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# Applicability of the De Marchi Hypothesis for Side Weir Flow in the Case of Movable Beds

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**Abstract:** Side weirs are widely used hydraulic structures typically designed and studied in the case of fixed bed conditions. In the case of subcritical flows, the hydraulics of side weirs can be modelled by using the classical De Marchi hypothesis. In the present work, a generalization of this hypothesis is developed for the case of side weirs over movable beds. Experiments showing the effects and feedbacks between the spilling discharge and the bed morphodynamics are presented. The application of the experimental observations to the generalized De Marchi hypothesis clearly show that the functioning of side weirs on a movable bed can be modelled by using this hypothesis. These findings could be instrumental for the design and verification of these structures. DOI: 10.1061/(ASCE)HY.1943-7900.0000566. © 2012 American Society of Civil Engineers.

**CE Database subject headings:** Weirs; Experimentation; Channel flow; Movable bed models.

**Author keywords:** Weirs; Subcritical flow; Experimentation; Channel flow.

## Introduction

Side weirs are hydraulic structures widely used to divert flow from a channel to prevent the downstream flow capacity of the channel that is exceeded, or to feed lateral channels particularly when sediment-free flow is required. Typical examples are the lateral spillways designed for flood protection of downstream river reaches or the storm water overflow in sewer networks used to divert excess flow that would otherwise overload the sewer system downstream.

The design of a side weir must take into account the mutual interaction between the head-discharge relationships of the channel and of the weir. When fixed bed conditions are considered, the head-discharge characteristics are essentially controlled by the geometry of the side weir, such as the crest length and height, and by the hydraulic features of the channel, i.e., slope, bed roughness, cross section. On the other hand, in movable beds the head-discharge relationships are invariably influenced by sediment transport processes, which determine the bed morphodynamics and channel roughness changes. As a fraction of the liquid discharge is diverted from the main channel, the downstream sediment transport capacity decreases. Thus, an aggradation process is established along the side weir modifying the bottom elevation and, consequently, the design efficiency. Furthermore, bedform dynamics induce changes in flow resistance, which in turn affects the rating

curve of the main channel. Hence, bed mobility may become an important factor affecting the performance of a side weir when it is located in an alluvial river. Unfortunately, a deep insight about the interaction between river bed evolution and side weir overflow is still lacking.

In recent years, many contributions on side weir flow have been proposed but most of them are referred to fixed bed conditions (Nadesamoorthy and Thomson 1972; Ranga Raju et al. 1979; Hager 1987; Bremen and Hager 1989; Borghei et al. 1999; Venutelli 2008).

In the literature, a few contributions concerning movable beds come from Rosier et al. (2005), who performed experimental investigations to analyze the interaction between side weir and main channel flow in a sand bed, and Catella et al. (2007), who analyzed the effects of bed mobility on side weir efficiency by modeling the morphodynamic evolution.

In the present study, the applicability of the well-known De Marchi hypothesis (De Marchi 1934) is assessed to predict the functioning of side weirs in the case of mobile beds. To achieve this aim, a set of experiments were carried out changing the height and length of the side weir and the input flow discharges. Experiments show that the lateral outflow produces both a bed aggradation downstream of the side weir and a bed degradation upstream of the weir. Such morphology alters the flow discharge diverted by the side weir with respect to the fixed bed case.

## Experimental Setup and Measurements

The experiments were conducted in a glass-sided rectangular flume of 0.44-m width, 5-m length and 0.30-m depth. The flume was subdivided longitudinally into two separate channels by a vertical glass wall. The first was the main channel, the width  $B$  is 0.3 m, representing the actual testing facility including the mobile bed ( $D_{50} = 0.84$  mm) and the side weir on the left bank. The second, 0.13-m wide, constituted a lateral channel allowing the evacuation of the diverted discharge.

At approximately 2 m from the flume entrance, a window of adjustable size, both in length and height, was inserted on the

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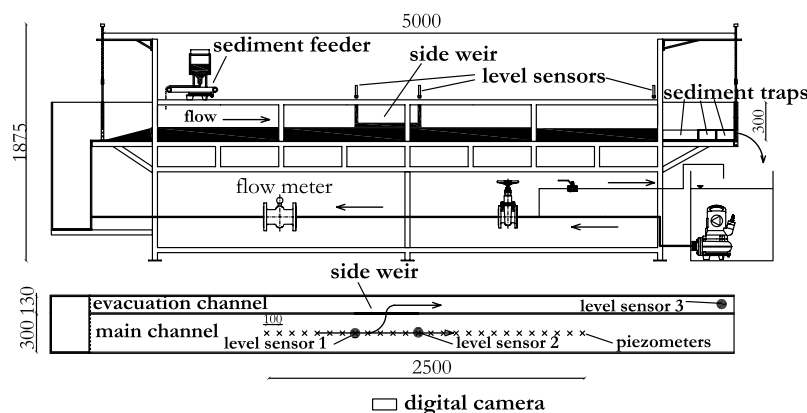


Fig. 1. Experimental setup: (a) lateral; (b) planimetric view of the flume (measurements are given in mm)

vertical separation wall, permitting the lateral overflow. The general layout of the experimental setup is shown in Fig. 1.

The recirculating water discharge was regulated by a valve and measured by means of an electromagnetic flow-meter (model Asamag, flow range 0–14 L/s). A sediment feeder at the channel entrance provided the input bed load  $Q_b$ . A sediment trap at the end of the channel allowed the conveyed material to be collected and reintroduced into the upstream sediment feeder.

The water free surface elevation was measured by means of a set of 26 piezometer intakes placed under the granular layer (see Fig. 1). For each experimental run, piezometers were read twice (once the equilibrium steady states were attained) obtaining 52 sets of water depth values along the measuring reach, referring to the channel bottom surface. The water free surface at the upstream and downstream side weir cross sections was recorded continuously in the centerline of the main channel by using two ultrasonic gauges located at the beginning and at the end of the side weir (see level sensors 1 and 2 in Fig. 1). The side overflow discharge was determined by using a sharp-crested weir located at the end of the evacuation channel and equipped with an ultrasonic gauge (see level sensor 3 in Fig. 1). As far as the monitoring of the bed morphology is concerned, a digital camera facing the weir was employed continuously to obtain data on bed channel variation. Images were then analyzed to obtain the time evolution of the longitudinal bed profile. Moreover, bed longitudinal profiles along the channel centerline were obtained by means of an ultrasonic gauge at the beginning and at the end of each experimental run. In a reach of about 1.4 m centered on the side weir, bed longitudinal profiles were also obtained along the left and right banks.

Each experiment consisted of two phases. In the first phase, the lateral window between the main channel and the evacuation channel was kept closed, and no lateral overflow occurred. An initial bed morphology was created by allowing the movable bed to self-adjust to the imposed liquid  $Q_u$  and sediment discharges  $Q_b$ . A vertical sluice gate placed at the main channel exit was used to regulate flow depth to obtain average uniform flow conditions in the main channel. In these conditions, the bed did not experience any variations (erosion and/or deposition) in the period of observation of approximately 1 h; this was checked by comparing the longitudinal bed profile obtained from the digital camera. Once equilibrium, i.e., a steady state condition with the bed not changing over time, was reached (hereafter named ‘equilibrium phase 1’), the lateral window was suddenly opened and the lateral overflow began. At the beginning of the second phase, the diverted liquid discharge  $Q_s$  rose during bed morphologic evolution for almost every experimental run, until a steady value was reached. In this second

phase, the experimental run continued until a new equilibrium steady condition for the new geometric configuration was achieved (hereafter named ‘equilibrium phase 2’). In both phases, the equilibrium conditions were assumed when successive measurements of the relevant physical parameters provided approximately constant values over a period of time of 1 h.

The experimental conditions are the following. The upstream discharges  $Q_u$  ranged from 2 to 12 L/s, the weir crest length  $L_w$  was taken as 23.5, 25.5, and 34.5 cm, the weir height  $p$  at equilibrium phase 1 was between 1.9 and 5.6 cm. In principle, each experimental condition corresponded to a set of two runs: one with an initial bed slope of 0.12% and one with 0.18%. In six experiments with the lower initial bed slope, the bedload discharge fed into the flume  $Q_b$  was set to zero and the initial bed slope was set to a smaller value of approximately 0.1%. A total of 44 runs were carried out with durations ranging from approximately 5 h to approximately 9 h.

Fig. 2 illustrates some of the relevant quantities, referred to in the following analysis, measured during the experiments at the equilibrium phase 1 and phase 2. In particular, during ‘equilibrium phase 1’ and in addition to the inflow and sediment discharges, the measured quantities are the side weir height  $p$  and the uniform flow depth  $y_u$ . During ‘equilibrium phase 2’, the measurements were for

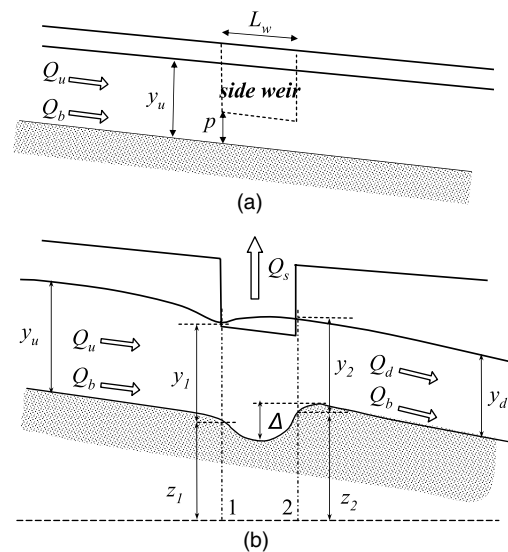


Fig. 2. Relevant experimental quantities measured during: (a) equilibrium phase 1; (b) equilibrium phase 2

the flow depths taken along the channel centerline, at the entrance  $y_1$  of the side weir, and the flow depth in the downstream reach  $y_d$ . Moreover,  $\Delta$  is the bed step height below the weir along the left longitudinal profile, and  $Q_s$  is the spilled discharge. Note that in all the experiments, the flow, in the reaches upstream and downstream of the weir, appears subcritical; for the flows at equilibrium of phase 1 the Froude number  $Fr_u$  was in the range (0.29, 0.50); whereas, for equilibrium of phase 2,  $Fr_d$  was between (0.27, 0.65). In some experiments, the flow near the weir was rather 3D (Neary et al. 1999) and, therefore, of difficult interpretation; however, on average, it can be reasonably assumed that the weir always operated in the case of subcritical flows.

## Experimental Results

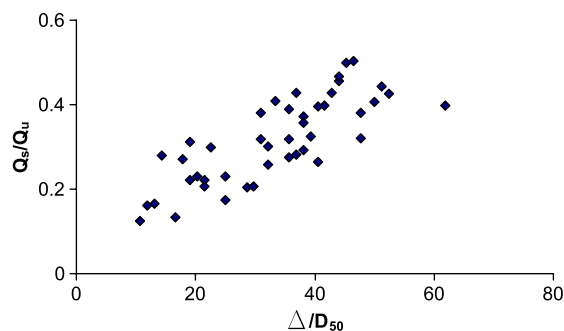
Results show that the lateral outflow induces both local bed deformations in front of the weir and generalized deformations in the reaches upstream and downstream of the weir.

In the channel reach facing the side weir, bed morphology typically appears strongly three-dimensional (see Fig. 3). Downstream of the weir, an aggradation phenomenon can be observed associated with an increase in bed slope. This higher bed slope allows the flume to convey the imposed solid discharge fed into the flume  $Q_b$  with the laminated liquid discharge ( $Q_u - Q_s$ ). Upstream of the weir, the accelerated free surface profile triggered by the drawdown effect of the weir (Montes 1998) induces a general degradation.

One of the most relevant quantities measured during the experiments is the flow discharge  $Q_s$  that spilled over the side weir at 'equilibrium phase 2'. In Fig. 4,  $Q_s$ , made dimensionless with the input flow discharge  $Q_u$ , is plotted as a function of the dimensionless bottom step height defined as  $\Delta/D_{50}$ . It appears that  $Q_s/Q_u$  increases with  $\Delta$ , thus suggesting that, as might be expected, increasing bottom step heights produce overall higher water levels on the side weir and higher values of the spilled discharge. Also, note that when  $\Delta$  tends to vanish, the spilled discharge also seems to vanish. These findings seem to suggest the key role played by the bed deformations on the functioning of the side weir. To better understand the increase in the spilled discharge in movable bed conditions, additional measurements regarding the flow pattern near the weir are required. These developments are currently in progress.



**Fig. 3.** Bed deformations in front of the side weir at the end of one experiment (the dashed area indicates local bed aggradation)



**Fig. 4.** The dimensionless flow discharge spilling over the side weir at equilibrium phase 2 versus the dimensionless bottom step height

## Assessment of the De Marchi Hypothesis

The method proposed by De Marchi (1934) is based on the assumptions that the specific energy of the flow in front of the weir may be taken as constant, and that the discharge along the weir can be computed by means of the same type of equation as in the frontal weir. The method proposed by De Marchi (1934) has been verified by several experimental investigations (Montes 1998). However, this method does not seem to have received confirmation of its validity when movable bed conditions are considered.

With the notation given in Fig. 2, the De Marchi hypothesis applied to the cross sections 1 and 2 yields the following:

$$z_1 + y_1 + \alpha_1 \frac{Q_u^2}{2g\Omega_1^2} = z_2 + y_2 + \alpha_2 \frac{(Q_u^2 - Q_s^2)}{2g\Omega_2^2} \quad (1)$$

with  $\alpha$  = energy (or Coriolis) coefficient;  $\Omega$  = liquid cross section; and the subscripts 1,2 refer to the cross sections at the entrance and exit of the side weir, respectively.

In principle,  $z_1 - z_2$  expresses the difference in the mean bed elevation between cross sections 1-2, such a difference can be expressed as

$$z_2 - z_1 = k\Delta \quad (2)$$

where, coefficient  $k < 1$ . With Eq. (2), it is stipulated that the bottom step height just below the side weir is the key variable describing the bed deformations responsible for the discharge spilling. The reduction coefficient  $k$  accounts for the 3D character of the bed deformations in front of the side weir, and, as a result, only a fraction of the whole step height needs to be considered in the energy balance.

In the downstream reach, the variation of the flow depth appears to be negligible, such that  $y_2 = y_d$ .

The energy coefficients  $\alpha_1, \alpha_2$  take into account the effect of the nonuniform distribution of velocity in the channel. Depending on the flow characteristics in the proximity of the side weir, the non-uniform character of the velocity distribution at the downstream end of the weir can be very pronounced. For instance, in the case of a flow in a prismatic channel with fixed bed, May et al. (2003) recommended  $\alpha_1 = 1.15$  and  $\alpha_2 = 1.75$  without any specification about their dependence upon the side weir length. El-Kashab and Smith (1976) showed, from experiments on side weir flow, that  $\alpha$  attains increasing values along the side weir, reaching values as high as 2.5 in the case of weir lengths to the order of 4 times the channel width and for subcritical flows.

In the experiments presented in this paper, the side weirs can be considered 'short' as the longest is 1.15 times the main channel width; however, the three-dimensional bed deformation in the proximity of the side weir makes the flow deviate from the uniform



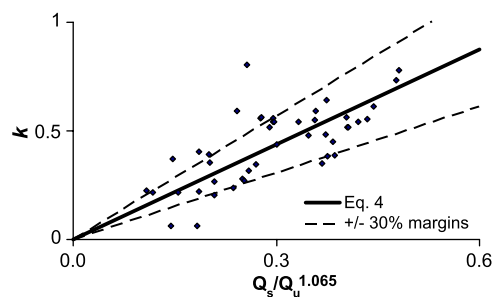


Fig. 5. The coefficient  $k$  for the reduction of the bottom step height

condition, thus suggesting that the effect of  $\alpha$  cannot be neglected. Therefore, following May et al. (2003), the following is stipulated:

$$\alpha_1 = 1.15; \quad \alpha_2 = 1.70 + DF \quad (3)$$

where  $DF = (y_u - p)/L_w * B/y_u$  is a dimensionless parameter taking into account the ratio of the water head over the side weir crest made dimensionless with the crest length and the ratio between the channel width and the mean water depth at equilibrium phase 1. Note that  $\alpha_2$  increases linearly with  $DF$ ; hence, the effects of non uniformity are assumed to be proportional to  $DF$ . In these experiments,  $\alpha_2$  ranges from 1.86 to 2.64.

Applying Eq. (1) to the experimental data, an estimate of the reduction  $k$  coefficient is obtained (Fig. 5). It appears that  $k$  correctly falls in the range  $0 < k < 1$ ; moreover, the parameter  $Q_s/Q_u$  seems to provide a reasonable description of  $k$  values. From data best fit regression and by using a power law type of formula, it appears that

$$k = 1.46 \left( \frac{Q_s}{Q_u} \right)^{1.065} \quad (4)$$

note that increasing  $Q_s/Q_u$  values are associated with stronger 3D bed deformations, which require higher values of the reduction coefficient  $k$ . Moreover, when  $Q_s$  vanishes,  $k$  correctly tends to zero as also the intensity of bed deformations become negligible.

To assess the De Marchi hypothesis, the specific energy, both at the upstream  $E_1$  and downstream  $E_2$  end of the weir is calculated as follows:

$$E_1 = y_1 + \alpha_1 \frac{Q_u^2}{2gB^2y_1^2}; \quad E_2 = k\Delta + y_2 + \alpha_2 \frac{(Q_u^2 - Q_s^2)}{2gB^2y_d^2} \quad (5)$$

with  $k$  evaluated with Eq. (4).

Results plotted in Fig. 6 show that in these experiments  $E_2 \approx E_1$ , therefore, confirming the validity of the De Marchi hypothesis even in movable bed conditions.

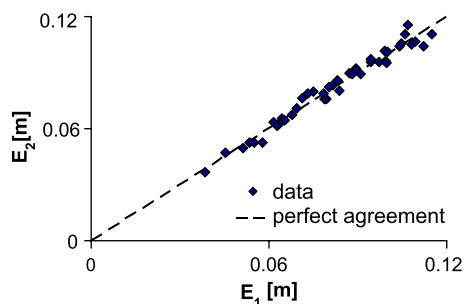


Fig. 6. The specific energy at the downstream end of the weir  $E_2$  against the upstream specific energy  $E_1$

## Conclusions

Side weirs are traditionally designed and studied in the case of fixed bed conditions. An analysis to investigate the effect of the mobile character of the bed on side weir functioning is presented in this paper. Laboratory experiments were performed in the common case of subcritical flow for several weir lengths and heights, and different input flow and sediment discharges. Results show that side weir flow deeply affects the bed morphology essentially giving rise to a positive step in the reach facing the weir as a result of a deposit in the downstream reach and a scour in the upstream reach. The experimental observations are interpreted within a rational framework developed in this study and on the basis of a generalization of the well-known De Marchi hypothesis in the case of movable beds. The experimental findings confirm the validity of this hypothesis even in the case of movable beds and provide a tool for estimating the spilled discharge for given bed deformations and reference flow specific energy.

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